PRELIMINARY EVALUATION OF THE USE OF COMPACTION PILES FOR IMPROVEMENT OF THE FOUNDATION SOILS OF THE COASTAL DIKES OF LAKE MARACAIBO, VENEZUELA

Bernardo Villegas, Project Engineer
Juan Murria, Consultant

MARAVEN, S.A., Apartado 173
Lagunillas 4016-A, Venezuela

A B S T R A C T

Recent studies have confirmed the liquefaction potential of the foundation soils of the coastal dikes of the Eastern Coast (Costa Oriental) of Lake Maracaibo, in western Venezuela, a region of moderate seismicity.

Mitigative measures consisting of downstream berms, with or without compaction piles, are being implemented. To evaluate the effectiveness of compaction piles against liquefaction of the silty sands and the improvement of low resistance clays, two field tests were performed, one in Bachaquero and one in Lagunillas.

The 0.60m diameter, 13m long piles were implanted in an 8 row, 5 column arrangement, 3m on centers. A ground levelling network was established. Piezometers, accelerometers and inclinometers readings were taken before, during and after pile implantation. The results obtained to date have confirmed that the berm and the compaction piles improve the properties of both the sandy and clayey soils and are, therefore, a valid method to improve the seimic stability of the coastal dikes of Lake Maracaibo.

INTRODUCTION

Subsidence associated with oil production in the east coast of Lake Maracaibo in western Venezuela was detected in the late 1920's (Murria and Jaeger, 1992). The geomorphology of the area (low, swampy lands) prompted the need to protect inhabitants and oil industry facilities from lake waters by means of coastal dikes.

The coastal protection system consists of 47 km coastal dikes, 59 km of inner (diversion) dikes, 90 km of drainage channels and 31 pumping stations with a combined installed capacity of 350.000 m3/h.

Western Venezuela is an area of moderate seismicity. Recent evaluation of the behavior of the coastal dikes under earthquake shaking have indicated the possibility of dike failure due to liquefaction of the foundation soils. Mitigative measures, consisting basically of downstream (landside) berms and upstream (lakeside) additional riprap, are being implemented. Post seismic stability analyses of different sections along the dike indicate required berms widths between 10 and 85m. Berm width is restricted in some areas by the presence of housing and/or oil facilities. It was decided for practical purposes to limit the berms width to 40m and to improve the foundation soils as needed.

The chosen improvement method has been stone compaction piles (Fig. 1). In order to evaluate the stone columns effectiveness, two pilot tests were performed, one in Bachaquero and one in Lagunillas (Fig. 2). The site selected for the first field test is located inland of the Bachaquero dike. It covers a rectangular area of approximately 1100 m2. The site was chosen because it is located in a very low risk area, away from any community or oil installations, and because the soil stratigraphy was considered typical of the areas where this methodology might be applied. The objectives of the Bachaquero pilot test were as follows (Sgambatti, Echezuria and Blanco, 1989):

1. Evaluate the effectiveness of the stone columns on the improvement of the geotechnical properties of the dike foundation soils,

2. Compare the effectiveness achieved by two different construction methods: vibro-replacement and casing-ramming,

3. Assess the influence of column spacing,

4. Study the effect of stone colums installation on the strength of clays and organic soils,

5. Evaluate pore water pressure variation during stone column construction and,

6. Evaluate time effects.

The site selected for the second field test is located on a dike section with 40m wide berm of the Lagunillas dike. (Fig. 2). The main reason to choose this area was the presence of high plasticity clay strata in the foundation soils.

The design of this field test was based upon the experience acquired from the Bachaquero pilot test. The objectives of the Lagunillas field test were the same as Bachaquero and, in addition, to avaluate the effect of stone column installation on the strenght of high plasticity clays.

Evaluation of soil site conditions at the different stages of the field tests was accomplished by means of standard penetration tests (SPT), cone penetration tests (CPT), in situ vane shear tests (VST), laboratory tests, and shear wave velocity determinations.

Pore pressure build-up during construction was monitored by means of piezometers installed at different depths. Additionally, movemments within the soil mass were monitored by inclinometers placed conveniently in the test areas. Ground deformation was monitored by means of a precise survey network.

BACHAQUERO PILOT TEST

The effect of strengthening the foundation soil by means of stone columns depends on many factors (Datye and Nagajaru, 1981), including:

- Method of construction and equipment characteristics,
- In-situ soil conditions, (soil type, gradation and fines content, relative density, state of stresses, and soil structure),
- Construction procedure and sequence of pile implantation and,
- Grid pattern and spacing between columns.

When programming the Bachaquero pilot test, some of these influencing factors were taken into account to permit later comparisons of their relative effectiveness. Two different construction techniques were tested: vibro-replacement and casing driving and ramming. To evaluate the effect of column arrangement, 2m, 2.5m, 3m and 4.0m c/c spacings were tested. An equilateral triangular array was decided upon as being considered the most efficient and economical.

A detailed testing program and an instrumentation set up were plannned for five different stages:

1. Stage 0: prior to any construction
2. Stage 1: after the construction of a 3m height embankment, which was used as a working platform,
3. Stage 2: immediately after columns construction,
4. Stage 3: two months after column construction and
5. stage 4: eight months after column construction.

A total of 120 stone columns were constructed by the two different constructions methods. Sixty by vibro-replacement and 60 by ramming-compaction. All the columns were constructed down to a depth of 16m, bearing on a firm silty clay. The nominal diameter varied between 0.55m and 0.60m. Actual diameters increased to as much as 1.20m in the very soft clay stratum.

During the construction of the stone columns, most of the instrumentation was badly damaged or rendered useless. Moreover, the inclinometer tubes were broken and filled with soil, indicating very large movements of the soil mass.

Site Stratigraphy

Table 1, below, summarizes the major soil strata encountered at the test site and their engineering properties.

TABLE 1. Summary of site stratigraphy

Stratum	Depth (m) From	Depth (m) To		Nspt	Su (Kg/cm2)	qc (Kg/cm2)
I	0	4-4.40	Very silty yellowish brown to dark greenish gray loose fine sand coarse very sandy non plastic silt, with inclusions of soft silty clay layers, locally organic.	13	-	50
II	4-4.40	8.20-8.70	Very soft to soft dark gray low plasticity silty clay and clayey silt.	1-2	0.22	4
III	8.20-8.70	9.10	Medium dense to dense yellowish to dark gray silty fine sand, which thins or pinchs out towards the south-western part of the area.	3-7	-	80
IV	9.10-10.0	13+	Soft to firm dark silty gray clay and clayey silt.	2	0.24	5

Construction Methods

Vibro-replacement Method

The vibro-replacement method is used to improve cohesive soils containing more than 18% passing # 200 U.S. standard sieve (Brown, 1976). The equipment used is the vibroflot which is sunk into the ground under its own weight, assisted by water or air jets as a flushing medium until it reaches the predetermined depth. The method can be used either with the wet or dry process. In the wet process, a hole is formed in the ground by jetting a vibroflot down to the desired depth with water. When the vibroflot is withdrawn, it leaves a borehole of greater diameter than the vibrator. The uncased hole is flushed out and filled in stages with 12mm-75mm size gravel. The densification is provided by an electrically or hydraulically actuated vibrator.

Casing Driving and Ramming Method

This method uses the energy delivered by a falling hammer to drive an open-ended hollow casing into the soil. The hole created by the lateral displacement of the soil, as the sleeve advances to the desired depth, begins to be filled with backfill material when the final depth is reached and the casing is progressively retrieved . The literature consulted so far (Datye and Nagajaru, 1981; Ghazali and Khan, 1986) indicates that there are no serious restrictions for its aplication to any type of soil, although this is not totally certain for cohesive soils.

Bachaquero Test Results

Soil Penetration Resistance

Evaluation was to be based on SPT and CPT results. However, CPT testing had to be discarded due to equipment problems. The only available comparable CPT data show that the vibro-replacement method increases penetration resistance of the granular strata even at 4m c/c column separation. Penetration resistance in the clayey materials does not seem to increase. On the other hand, the results corresponding to the casing driving and ramming method indicate that even columns spaced 4m c/c increased the penetration resistance of both cohesive and granular soil in the range from 100 to 250% and the cohesive soils increased their penetration resistance in a range from 25% to 50%.

The data obtained from Nspt values, comparig results previous to column construction with those approximately two months after column construction, indicate a considerable increase in penetration resistance in the area where the driving and ramming method was used, whereas the results show a moderate improvement in Nspt values in the area where the vibro-replacement method was used.

Shear Strength from Field Vane Tests

The data obtained from in-situ vane tests show that both construction methods increase soil shear strength. Recorded values for both methods are very similar for either peak or residual strengths. However, the scatter in the vibro-replacement test is much larger.

Peak and residual strength values after treatment tend to be lower (around 6m to 7m and near 10.5m to 11.5m depths) for the two construction techniques. Strength values in the vibro-replacement area are lower than those in the casing driving and ramming area. This may be a consequence of the level of consolidation reached by the clay at the time of performing the field vane tests.

Shear vane results in the vibro-replacement area indicate that strength values are larger in the zones adjacent to sand stratum for organic silt layers. The improvement achieved in the cohesive soils with both methods was in a order from 100% to 40%.

LAGUNILLAS FIELD TEST

Based on the Bachaquero pilot test results the following was decided for the Lagunillas field test:

- Adopt the casing driving and ramming method
- Keep the 0.60m nominal pile diameter and adopt a 13.5m pile length
- Adopt the triangular array 3m c/c

The Lagunillas field test involved: (1) determination of the original site soil conditions, (2) installation of the instrumentation to monitor pore water pressures, ground acceleration and ground deformation, (3) construction of 20 stone columns and (4) determination of the "after column construction" soil properties. The columns were constructed down to a depth of 13.5m, bearing on a firm high plasticity clay.

The field work was programmed to be performed in three phases, as follows:

- Determination of the "before" soil conditions and installation of the instrumentation,
- Construction of stone columns and monitoring of the construction process
- Determination of the "after column construction" soil conditions

Site Stratigraphy

The site stratigraphy consists of alternate layers of granular, organic and cohesive soils. Minor textural and color variations and inclusions of other soil types within each generalized stratum are present.

Major strata of the site stratigraphy are summarized in Table 2, below:

TABLE 2. Summary of site stratigraphy

Stratum	Depth (m) From	Depth (m) To	Soil Description
I	3.35	6.50	Medium dense, very sandy plastic silt (MNP)
II	6.50	7.95	Medium compact, low plasticity silt (ML).
III	7.95	12.15	Very soft to soft, low plasticity silty clay (CL).
IV	12.15	14.40	Stiff, high plasticity clay (CH)

Test Program

Soil improvement was evaluated by means of penetration tests (SPT and CPT), in-situ vane shear tests (VST) and laboratory tests conducted on undisturbed samples. All the in-situ tests have been conducted in four stages as follows: inmediately after columns construction (3-7 days), one month later, three months later, and eleven months later. The in-situ tests program is shown in Figure 3. Results from laboratory tests are not presented in this paper.

Test Instrumentation

The pore pressure behaviour before, during, and after the columns construction were monitored by electrical and vibrating wire piezometers at different depths.

The surface movements were monitored through precise surveying of object points. The location of the instrumentations is shown in Fig. 3.

Lagunillas Test Results

Pore Pressure Build Up

The construction sequence (Fig. 4) has an effect on the generation and dissipation of pore water pressures. The relative increments with respect to the pressure before construction have an irregular pattern, which is believed to be a consequence of partial dissipation and redistribution of pore pressures due to the construction period.

Soil Penetration Resistance

Fig. 5 illustrates the corrected average values of Nspt for stages zero and 4. Fig. 6 presents the trend of the cone resistance (qc) for different stages. These figures indicate a considerable increase in penetration resistance. This increase ranges between 30% and 300% in the first stratum with the largest values toward the upper portion of the layer. Only slight increments in penetration resistance were obtained in cohesive soils. Nonetheless, penetration resistance tests are not sensitive enough to evaluate strength changes in cohesive soils.

Shear Strength from Field Vane Tests in Cohesive Soils

A series of in-situ vane tests were conducted to monitor the variation in Su with time after construction. In order to interpret the results, it was necessary to examine general trends rather than specific details since the soils at the test section vary considerably across the area.

The increase of peak shear strength values range between 30% and 60%. The residual shear strength values presents an increment between 40% and 100%. Fig. 7 shows the trends of the field vane tests results.

Monitoring of Ground Deformation

Based on the experience from the Bachaquero pilot test, three control point were placed away from the test section in order to guarantee their stability (Pedroza et al, 1992). Twenty two object points conformed the survey network, conveniently spread within the test section to provide a complete picture of the ground deformation (vertical and horizontal) due to stone column construction. (Fig. 8).

Horizontal Displacements

Field data were reduced to compute adjusted coordinates of object and control points. This adjustment provided displacement vectors of each point and its associated error ellipse at 95% confidence level. Significant vectors are shown in Fig. 9. The results shown by these vectors indicate the trend of the body displacement due to pile construction. The maximun horizontal displacement measured was of the order of 0.7m. As expected, some points returned to their initial position once the stone columns were constructed, thus confirming the elastic component of ground deformation.

Vertical Displacements

A total of 17 campaigns of high precision first order levelling were performed using two bench marks as controls located near the test section. The reduced height differences from the field measurements were introduced in a graphic computer file for continuous evaluation. An increment of height was obtained in the first five campaigns until construction stopped, with a total displacement of 0.076m. A height decrease was monitored later on with a total displacement of 0.156m. (Fig. 10).

It was, therefore, shown that eight months after the construction, berm vertical displacement had not stopped.

CONCLUSIONS

- In the Bachaquero pilot test the casing driving and ramming method showed the most effectiveness.

- The geotechnical instrumentation design for the Lagunillas field test was based on experience obtained from the Bachaquero pilot test. The entire design was conceived in order to monitor the pore water pressure build up and the surface movements.

- Results obtained from penetration tests and in-situ vane shear tests in the Lagunillas field test, show that the casing driving and ramming technique improves the engineering properties of both granular and high plasticity soils.

- The construction sequence appears to have an important effect on the pore water pressure generation. The highest excess pore pressure was registered in stratum III, during the casing driving operation.

- No increment of pore pressure due to stone columns construction was registered 10-12 meters away from the piezometers.

- Vertical displacements, as expected, showed heave during stone column construction. When construction stopped, ground level started to settle down to values even lower than the initial ones. This indicates soil compaction after the appropiate time period.

- Horizontal displacements showed the pattern followed by the berm, when subjected to strong compaction of the soil. This means that the berm deformed landwards.

REFERENCES

- Blanco, C. Villegas and Sgambatti, J. (1992), "Compaction Columns Field Tests in Heterogeneous Soil Profile", Proc. 3rd. Int. Conf. Case Histories in Geotechnical Engineering., St. Louis.

- Brown, R. E. (1976), "Vibroflotation Compaction of Cohesionless Soils", ASCE Nat. Water Res. Conv.

- Datyl, K.R. and Nagajaru, S. S., "Design Aproach and Field Control for Stone Columns", X Int. Conf. on Soil Mec. and Found. Eng., Stockolm.

- Ghazali, F. and Khan, A. (1986), "Granular Soil Improvement, Some New Physical Methods for Saudi Arabian Projects", Journal of Engineering and Applied Sciences, Vol 3, pp 113-123.

- Murria, J., Leal, J. and Jaeger, L. (1992), "Monitoring and Modelling Ground Subsidence in Western Venezuela Oilfields: An Update", 6th. International FIG-Symposium Deformation Measurements, Hanover Germany.

- Pedroza, M., Villegas, B., and Murria, J. (1992), "Monitoring of Ground Deformation Caused by Stone Pile Construction to Mitigate Liquefaction Potential, Costa Oriental Dikes, Venezuela", 6th. International FIG-Symposium Deformation Measurements, Hanover Germany.

- Sgambatti, J., Echezurfa H. and Blanco C. (1989), "Compaction Pile Field Test on a Heterogeneous Soil Profile, Unpublished INTEVEP report, Los Teques.

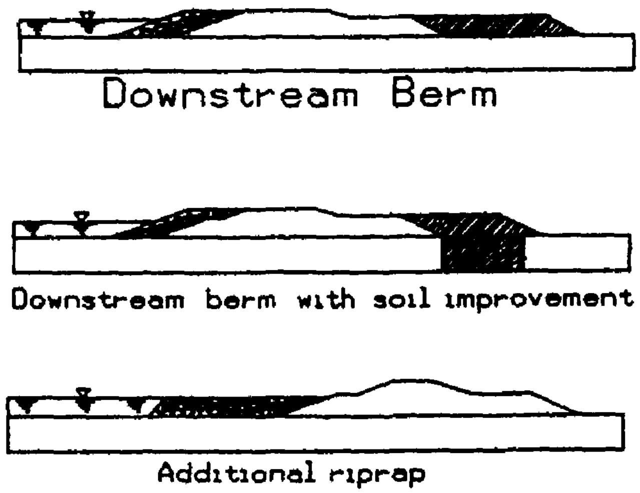

FIGURE 1. MITIGATIVE MEASURES

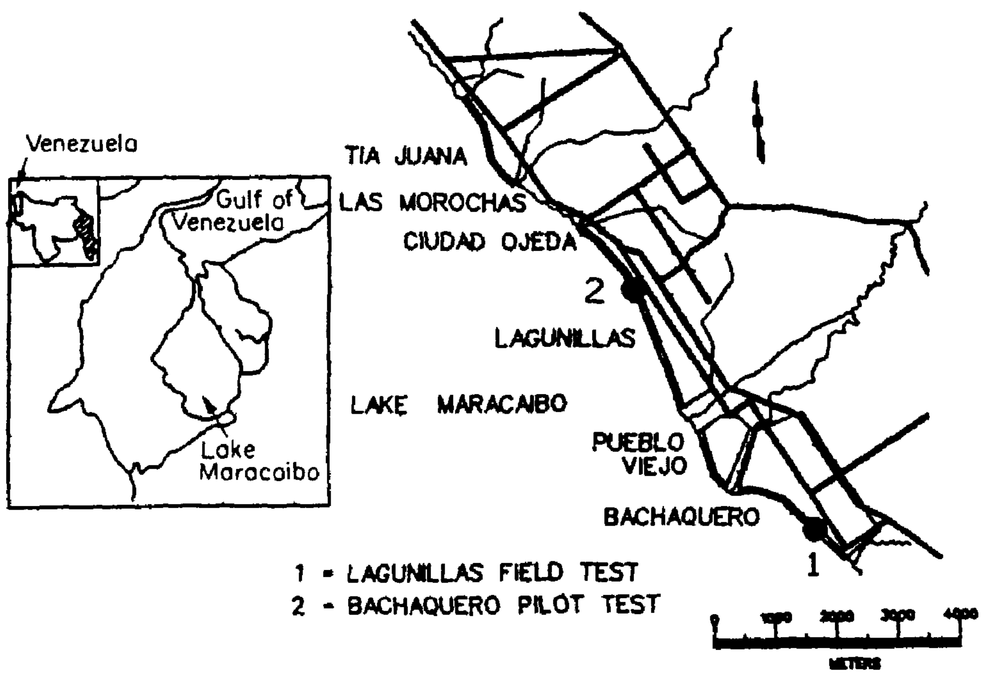

FIGURE 2. LOCATION OF FIELD TESTS

SYMBOLS	DESCRIPTION	STAGES P0	P1 DIA 3-7	P2 DIA 30	P3 DIA 90	P4 DIA 320
	BORING AND WASHING FOR SHELBIES SAMPLES	2	2	-	-	2
	C.P.T.	5	5	-	2	2
	S.P.T. V.S.T.	1 3	2 9	1 3	2 6	2 6
	PERMEABILITY	2	2			
	CROSS HOLE	1				
	OPEN PIPE PIEZOMETERS	8				
	DYNAMICS PIEZOMETERS	3				
	ELECTRICS PIEZOMETERS	4				
	ACCELEROMETERS	10				
	INCLINOMETERS	4				
	HORIZONTAL-VERTICAL CONTROL DISPLACEMENTS	22				
	HYDRAULIC FRACTURE TEST					3

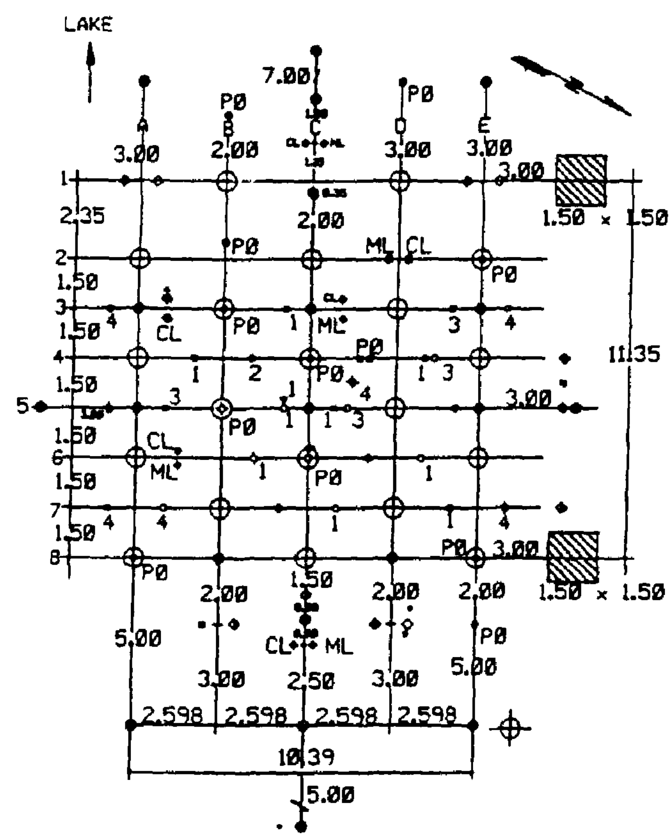

FIGURE 3. LAGUNILLAS PILOT TEST LAYOUT OF INSTRUMENTATION

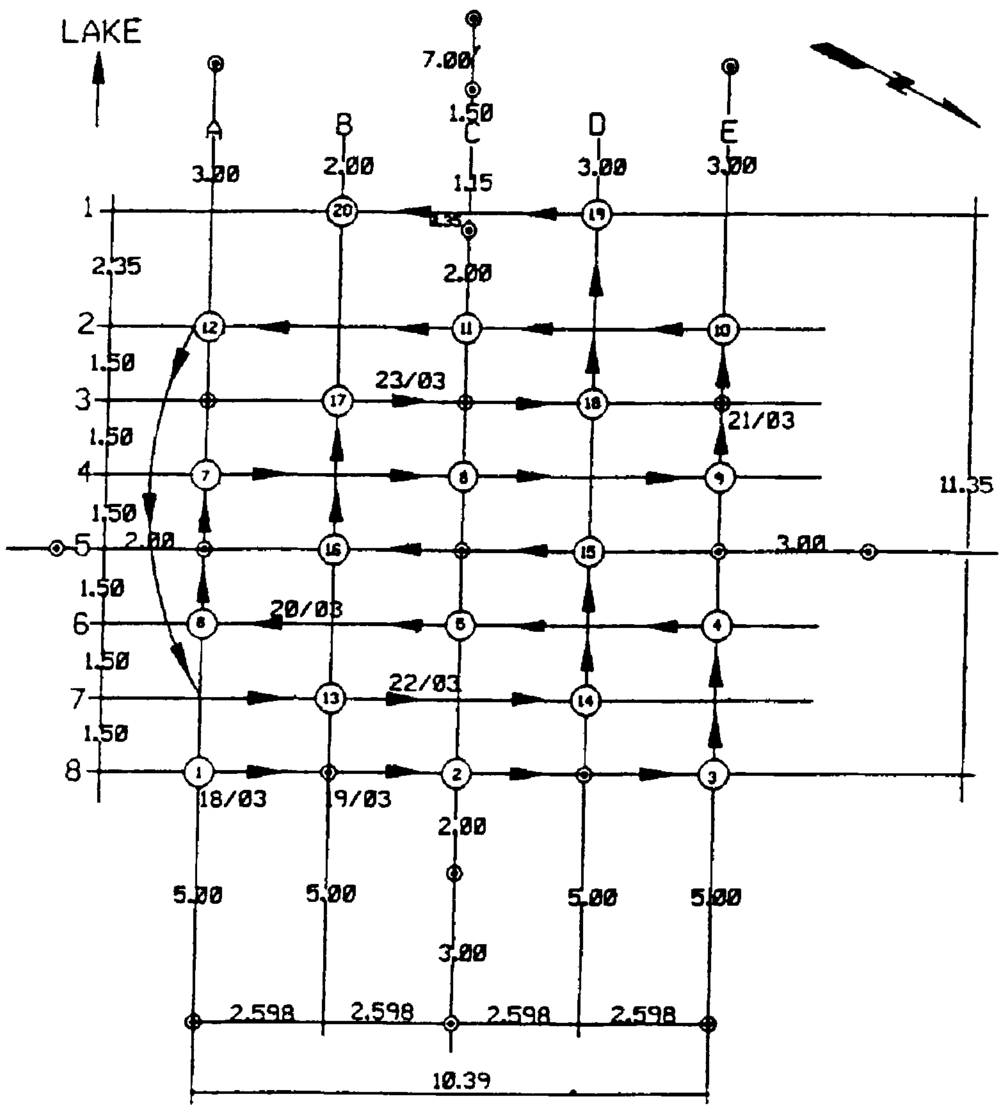

FIGURE 4. LAGUNILLAS TEST COLUMN CONSTRUCTION SEQUENCE

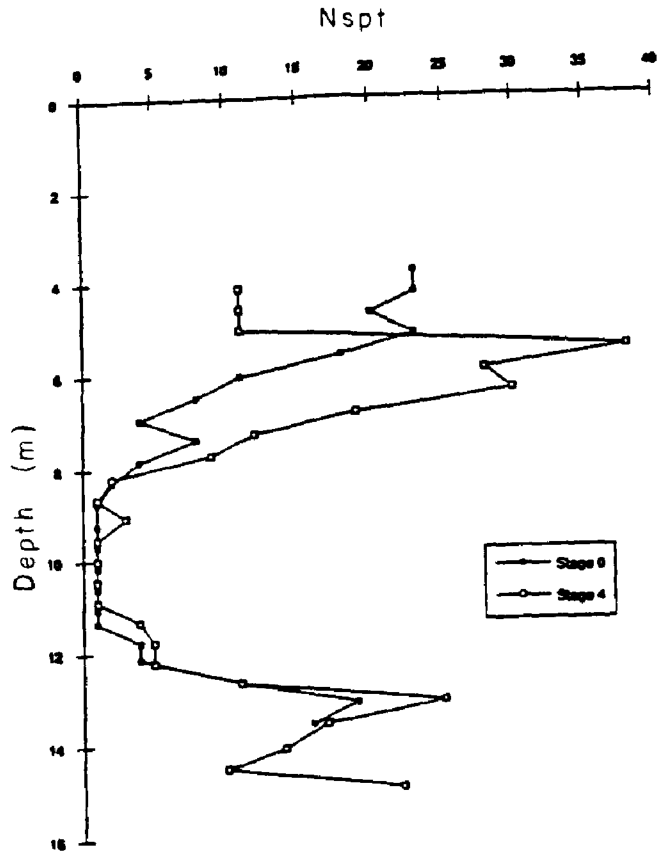

FIGURE 5. SPT RESULTS, STAGES "0" AND "4"

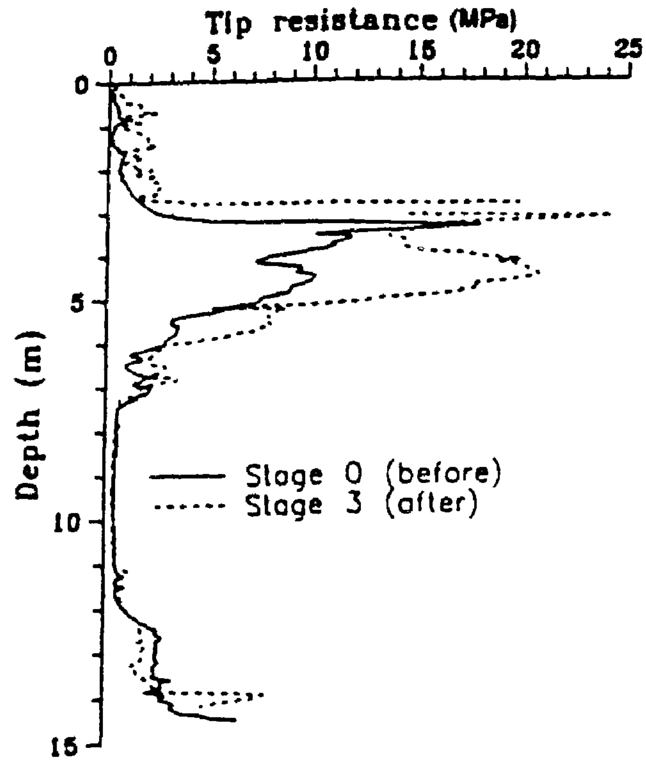

FIGURE 6. CPT RESULTS, STAGES "0" AND "3"

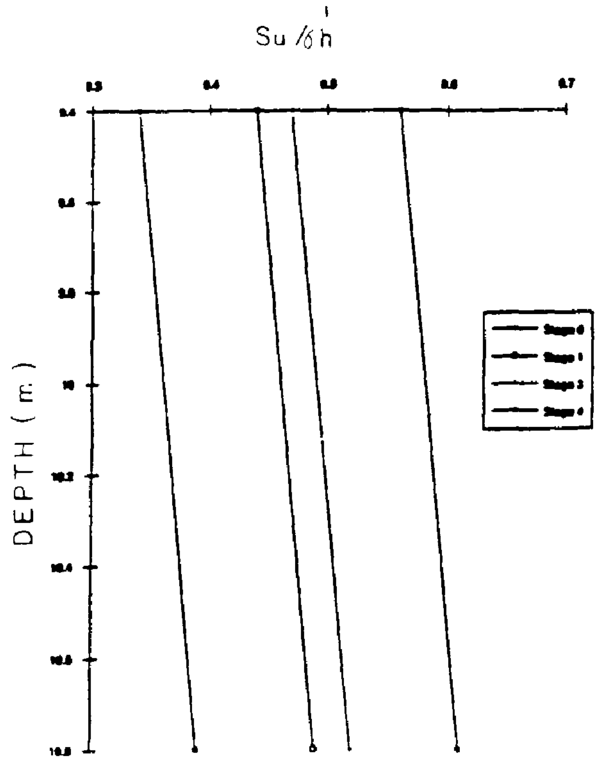

FIGURE 7. UNDRAINED SHEAR STRENGTH TREND AT VARIOUS STAGES

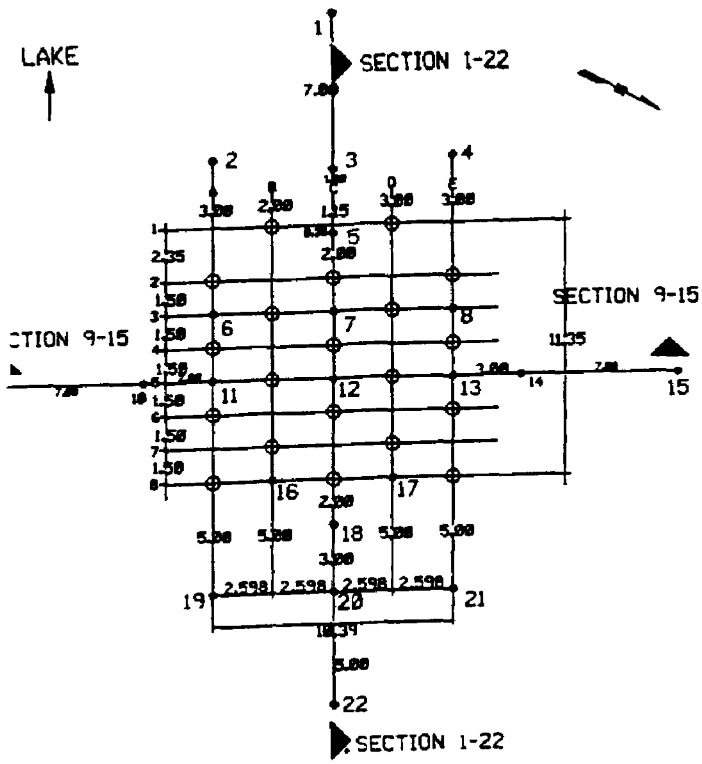

FIGURE 8. LOCATION OF OBJECT POINTS 1 TO 22

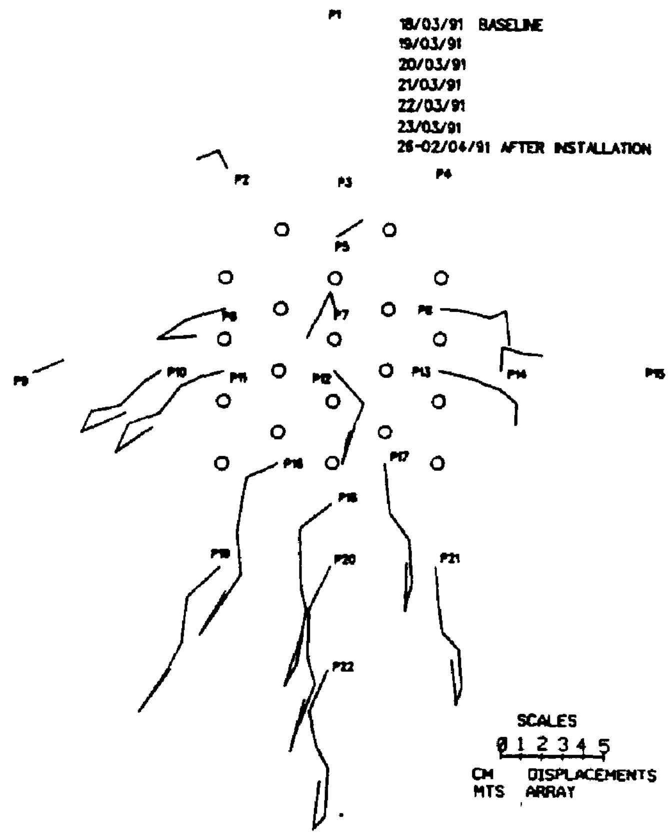

FIGURE 9. HORIZONTAL DISPLACEMENT VECTORS

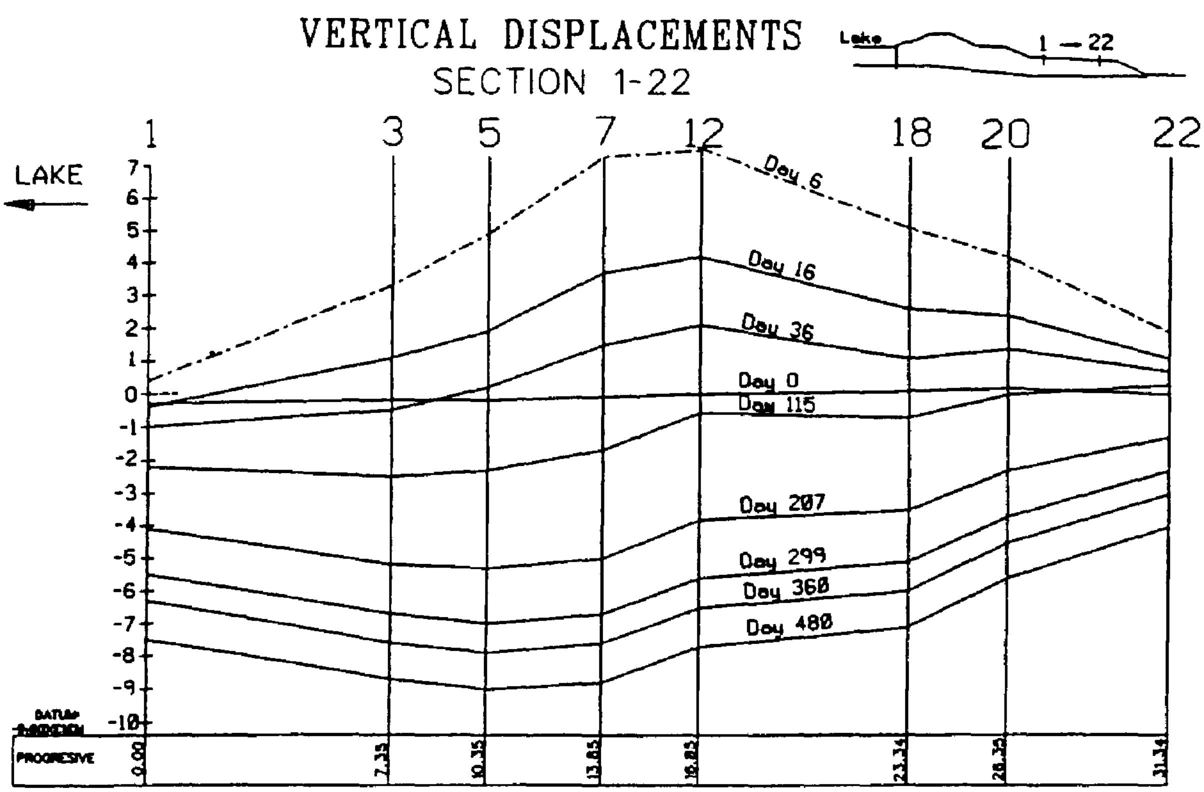

FIGURE 10. VERTICAL DISPLACEMENTS ALONG SECTION 1-22

V LIFELINE PERFORMANCE AND LIQUEFACTION DURING EARTHQUAKES

MINIMIZATION OF SEISMIC DAMAGES

K. KUBO

It is the sincere wishes of all the people, and also just the dream of human kinds to wipe out any kind of natural disaster, including seismic damages from the earth. However, the real situation is far from our dream. For an example in Japan, a volcanic eruption of Mt. Fugen has been damaging villages on its foothill and is forcing people to lead the long and much uncomfortable refugee lives. Recently, elevated highway bridges were heavily damaged, and gas- and water pipes were broken in the event of Northridge earthquake. People in Southern California are badly threatened due to a couple of strong earthquakes which have occurred in succession since 1971 San Fernando Earthquake, and civil engineers are now very anxious about advent of new type of damages to building and civil engineering structures.

United Nations Educational and Scientific and Cultural Organization (UNESCO) had Inter- governmental Conference on Seismology and Earthquake Engineering in Paris in 1964, aiming the extermination of seismic damage and the construction of the safer national land with such good cooperative contribution of earthquake engineers as international exchange of technical knowledge. In spite of UNESCO effort, strong earthquakes have attacked many times, and a lot of human lives and properties have been lost so far. In 1976 UNESCO again convened the Intergovernmental Conference for mitigating seismic damages, where conference members paid special attention to Haisheng earthquake of Feb. 4, 1975, M=7.3, in China, as it was successfully predicted and people in Haisheng area were forced to evacuate five and half hour in advance the occurrence of the earthquake. In the conference, most of the attendant were thinking that earthquake prediction would be a main resolution for mitigating seismic disasters. However, in the next year, Tangshang earthquake broke out and most of brick apartment houses collapsed and it is said that about 400,000 peoples were killed and injured. So far, only one earthquake and no more has been successfully predicted, and recently discussion items of many seismologists are focussing on the difficulty of earthquake prediction. For reducing seismic damage as low as possible, efforts of earthquake engineers to design and construct earthquake-proof structures are indispensable highly evaluated.

In Japan, there were 36 earthquakes between Nohbi earthquake, 1891, M=7.0 and Hokkaido-Nansei-Oki earthquake M=7.6, 1993, that means that our country has been attacked by earthquakes every about 3 years. Fortunately, no earthquake with so many victims has not occurred since the Fukui earthquake, 1948, where more than 1000 people were killed, but there were several earthquakes causing several thousands, or more than ten thousands victims, in underdeveloping countries.

The writer would like to point out 4 reasons as stated in the followings, for why seismic damage can not be wiped out, 1) Structural design is based on principles of economics, 2) Many kinds of unsolved factors ruling structural seismic damages, 3) Occurrence of new damage patterns which have not been investigated clearly so far, 4) Existence of many new construction materials and structures, which have not baptized by severe earthquakes.

It is surely possible for us to design and construct no-damaged structures even though they were shaken by very severe earthquakes, if no principle of economics would be taken into consideration. Generally speaking, it is of very low possibility for a structure to be shaken by destructive earthquakes during its life time, and the cost which is needed to make overstrung structures is rarely paid unless the structure experiences the strongest earthquakes, and shows its earthquake-proof.

It is well-known that ground conditions have the direct effect on degree of seismic damage to structures, but there are very few cases, in which seismic design coefficients are changed from on a structure to another one, based upon precise test results of soil conditions, and dynamic characteristics of each structures. It is concluded that structures designed by the principle of economics will be sustained seismic damage.

In the event of Northridge earthquake, Jan. 17, 1994, peak ground acceleration as much as 2000 gal was recorded on free field in Tarzana region, and the strong motion seismo graph installed in Kushiro meteorology indicated 922 gal as the maximum value during Kushiro-Oki earthquake, Jan. 15, 1993. In spite of very high ground acceleration, there were low percentage of damage ratio in San Fernando valley as well as in Kushiro City, even though there were seismic damages to buried lifeline pipes.
Since 1915, when static seismic coefficient design method was proposed by Prof. R. Sano, degree of seismic damages to structures is said to be dependent on the maximum value of peak ground acceleration, but this scenario has become useless, and seismic damage to structures can not be discussed only by peak ground acceleration. The value of 2000 gal has never been recorded in the past earthquakes and therefore this is very shocking for designers of nuclear power plants.
In the event of Niigata earthquake 1964, RC buildings subsided, tilted or overturned, and man-holes floated up as sandy ground in Niigata City liquefied widely, as well as perfectly, and also bridge piers were shifted horizontally or buried pipes sustained large deformation due to lateral spreading of liquefied ground, whose maximum movement reaches about 12 meter.

In 1978, Sendai city experienced an earthquake of M=7.4. The epicenter of which was about 100 kilometers away from the city center, The specific damage was to the pipe joints of small diameters. It is clear that collar rings of threaded pipes can be easily deformed, causing loose connection under repeated axial loading, not withstanding the fact that arc-welded steel pipes perform very well and survived without any damage in Niigata earthquake. In the event of Kushiro earthquake, 1993, damage ratios for threaded small diameter steel pipes of the Kushiro Gas Co. were very high in comparison with the ratios of the other kinds of pipe joints.

It is concluded that in future, there remains some possibility of occurrence of the damage performances of structures which can not be anticipated scientifically, and severe failures of buildings as well as civil engineering structures.

Another reason for which seismic damages to structures can not be eradicated is like that such new types of structures as high-rise buildings or very long bridges and new construction materials, for an example high-tensile steel and welded joints are now widely applied to various kinds of structures, whose performance during strong earthquakes has not yet fully checked and verified. It is needless to say that their earthquake-proof is corroborated by using up-to-date analytical method, and laboratory test results, but it can not be denied for civil engineers to have vague anxiety, as these structures have never baptized by strong earthquakes.

It is said that anti-seismic design coefficient of 0.06 specified in 1950 s was enough for buildings of regular shapes and constructed with good workmanship. Seismic coefficient of the same value has been used for buildings of irregular shapes as well as bridges, and therefore many seismic damages to building and civil engineering structures have occurred in the events of recent severe earthquakes. It is good lesson obtained through strong earthquakes that since 1971 San Fernando earthquakes, seismic coefficients for bridges has been pressed upon re-examination.

The most important research item of earthquake engineering is to design and construct structures of enough resistance against earthquakes, since the establishment of the Investigation Committee for Earthquake Disaster Prevention in 1892, one year after the Nohbi earthquake of 1891. This concept is still correct at present, by using which many researchers and engineers are devoting themselves for mitigating seismic damages.

As it is very difficult for us to eradicate seismic damages concretely so far, and a lot of money is needed for strengthening the existing vulnerable structures, new design concept has been spotlighted in which damaged structures and facilities are restored as early as possible, for reducing the stopping period of services.

It is very useful software countermeasure for gas engineers to divide the whole service area into many blocked districts before a future earthquake, for stopping gas-supply only in limited damaged districts, and for doing restoration works rapidly. Another kinds of software earthquake programs are quick collection (within several minutes) and dissemination of the earthquake information as well as damage ones, technical development of detecting damage patterns of lifeline buried pipes, preparation of restoration machines and equipments, medicine for emergency use, foods as well as drinking water, and disaster drill.

Both hardware technology and software countermeasures are considered to be very useful for mitigating seismic damages to human lives and properties and quick restoration of the lifeline facilities.